



Carbonation depth predictions in concrete bridges under changing climate conditions and increasing traffic loads

Chao Jiang^{a,b}, Xianglin Gu^{a,b,*}, Qinghua Huang^{a,b}, Weiping Zhang^{a,b}

^a Key Laboratory of Performance Evolution and Control for Engineering Structures, Ministry of Education, Tongji University, 1239 Siping Rd., Shanghai 200092, PR China

^b Department of Structural Engineering, College of Civil Engineering, Tongji University, 1239 Siping Rd., Shanghai 200092, PR China

ARTICLE INFO

Keywords:

Concrete
Carbonation
Fatigue damage
Climate change
Traffic loads

ABSTRACT

This paper first introduced a numerical carbonation model (NCM) for fatigue-damaged concrete. Then, based on the NCM, a simplified carbonation model (SCM) for fatigue-damaged concrete was established, comprised of a non-damaged concrete contribution term and a fatigue damage contribution term, through Monte Carlo simulations. Both the NCM and SCM were verified by experimental results reported in the literature. Subsequently, an incremental method was proposed to consider the time-variant fatigue damage and exposure conditions. Finally, a case study was conducted, which determined that cumulating fatigue damage could have a big influence on carbonation depth evolution. Meanwhile, differences in the cumulating fatigue damage could lead to noticeable differences in carbonation depth evolution among different points in the same concrete bridge. Moreover, drastic increases of CO₂ concentration and temperature could induce relationships of carbonation depths with square roots of service times to remarkably deviate from the proportional laws widely accepted under a time-invariant environment.

1. Introduction

Reinforced concrete (RC) bridge construction has been a priority in China for many years along with the rapid development of highway or high-speed railway networks. Under an atmospheric environment, these RC bridges not only suffer carbonation-induced steel reinforcement corrosion but also undergo fatigue damage caused by running vehicles or trains. Steel reinforcement corrosion usually causes RC bridges to be functionally deficient or even obsolete due to delaminating concrete covers and decreasing reinforcement cross-sections [1–11]. Furthermore, fatigue damage often induces micro cracks in concrete to coalesce into macro ones, which accelerates the carbonation process in concrete. Therefore, the influence of fatigue damage on concrete carbonation has drawn significant attention.

Several researchers [12–17] have conducted experiments to discern the effects of fatigue damage on carbonation. These researchers consistently found that both tensile and compressive fatigue damage can increase carbonation depths. However, in most cases, these experiments were designed to explore carbonation depth evolution with respect to exposure duration in fatigue-damaged concrete under constant exposure conditions. The effects of exposure conditions, i.e., temperature (*T*), relative humidity (*RH*) and carbon dioxide (CO₂) concentration on

carbonation depths in fatigue-damaged concrete were thus under-represented. Moreover, except for Jiang et al.'s experiment [13] which involved axial fatigue-damaged prism concrete specimens, the majority of these experiments [12,14–17] utilized fatigue-damaged plain concrete beam specimens. In plain concrete beams, the stresses in the compressive zones are comparable to stresses in the tensile zones, which are usually less than or equal to the tensile concrete strength f_t' . For concrete, the tensile strength f_t' usually ranges from 1/10 to 1/8 of the corresponding compressive strength f_c' . Consequently, compressive stresses in plain concrete beams, which range from 0 to f_t' (i.e., from 0 to $0.1f_c'$ or $0.125f_c'$), are much smaller than those ranging from $0.4f_c'$ to $0.6f_c'$ which commonly exist in real-world concrete structures. To remedy these deficiencies, the authors' group conducted a systematic experimental investigation on carbonation in prism concrete specimens damaged by uniaxial and eccentric compressive fatigue loads under multiple exposure conditions [18]. In that experimental investigation, three common fatigue damage patterns were identified, which were gradient compressive damage (GCD), gradient tensile damage (GTD) and uniform compressive damage (UCD). Residual strain and residual curvature were chosen as fatigue damage indexes to evaluate fatigue damage degree and fatigue damage gradient, respectively, in GCD, GTD and UCD concrete. The effects of fatigue damage, i.e., residual strain

* Corresponding author. Key Laboratory of Performance Evolution and Control for Engineering Structures, Ministry of Education, Tongji University, 1239 Siping Rd., Shanghai, 200092, PR China.

E-mail addresses: cjiang@yeah.net (C. Jiang), gxl@tongji.edu.cn (X. Gu), hqh@tongji.edu.cn (Q. Huang), weiping_zh@tongji.edu.cn (W. Zhang).

<https://doi.org/10.1016/j.cemconcomp.2018.07.007>

Received 25 December 2017; Received in revised form 2 June 2018; Accepted 16 July 2018

Available online 17 July 2018

0958-9465/ © 2018 Elsevier Ltd. All rights reserved.

and residual curvature, on carbonation of concrete were thoroughly analyzed and reported [18].

Despite the large number of experimental studies, few efforts have been made to model the carbonation process in fatigue-damaged concrete theoretically. Fatigue damage affects concrete carbonation through influencing the effective diffusion coefficient of carbon dioxide (CO₂) in fatigue-damaged concrete. The authors established and verified a numerical carbonation model (NCM) for concrete with tensile fatigue damage by incorporating a residual strain-based effective CO₂ diffusion coefficient [15]. In this paper, the NCM was extended to concrete with compressive fatigue damage by proposing a similar form of residual strain-based effective CO₂ diffusion coefficient (Section 2). The extended NCM was validated by comparing the model-predicted carbonation depths with experimental results from studies done by the authors [15,18] and other researchers [12,13,16,17] (Section 3).

Although the NCM can be used to accurately predict carbonation depths in fatigue-damaged concrete with various fatigue damage degrees and fatigue damage gradients under multiple exposure conditions, the numerical carbonation model is too complex for most civil engineers as well as prohibitive due to huge computational costs involved in solving partial differential equations. Hence, a simplified carbonation model (SCM) for fatigue-damaged concrete, which establishes an explicit relationship between the carbonation depth and the exposure duration, is needed for fast carbonation depth predictions in real-world projects. As such, this paper further developed a SCM that considered the effects of both tensile and compressive fatigue damage based on the NCM through Monte Carlo simulations (Section 4). This SCM comprised of a non-damaged concrete contribution term and a fatigue damage contribution term, which was also verified by experimental results reported in the literature. The simplified carbonation model can be used to predict the carbonation depth evolution in concrete with determined fatigue damage degrees and gradients under constant exposure conditions.

On the other hand, global climate change is altering the exposure conditions around RC bridges year by year [19] and, simultaneously, fatigue damage in RC bridges is accumulating with service time [20]. Thus, both exposure conditions and fatigue damage are time-variant. For that reason, if one wants to predict the carbonation depths in RC bridges under the context of global climate changes and traffic load increases, the simplified carbonation model alone is inadequate. Accordingly, based on the simplified carbonation model established in this paper, the authors further proposed an incremental method to consider the time-variant exposure conditions and fatigue damage (Section 5). Finally, a case study was conducted to observe carbonation depth developments with respect to service times at representative points of an RC bridge under the context of changing climate conditions and increasing traffic loads (Section 6).

2. Numerical carbonation model (NCM)

2.1. Carbonation equation

Although there are three different fatigue damage patterns, i.e., GCD, GTD and UCD (identified by the authors in Ref. [18], or see Fig. 1), modeling the carbonation process in fatigue-damaged concrete is similar to modeling that in non-damaged concrete as conducted by Papadakis et al. [21]. Fig. 1 presents the equation schematic for Eq. (1), based on the mass conservation of CO₂ in the fatigue-damaged concrete strip and Fick's diffusion laws. This made it easy to obtain the following partial differential carbonation equation for fatigue damaged concrete:

$$\frac{\partial}{\partial x} \left(D_d \frac{\partial [\text{CO}_2]}{\partial x} \right) = \frac{\partial}{\partial t} (\varphi(1 - S)[\text{CO}_2]) + Q \quad (1)$$

where, [CO₂] denotes the concentration of CO₂ in the gaseous phase of concrete pores (mol/m³). φ is the porosity of concrete, and S is degree

of saturation of concrete pores, both of which can be determined by the methods proposed by Papadakis et al. [22]; D_d and Q denote the effective CO₂ diffusion coefficient (m²/s) and the total reaction rate of carbonation (mol/m³s) in fatigue-damaged concrete, reflecting the physical and chemical aspects of the carbonation issue, respectively.

2.2. Effective CO₂ diffusion coefficient (D_d)

In our previous work [15], we established a residual strain-based effective CO₂ diffusion coefficient for concrete with tensile fatigue damage by choosing fictitious cracks to represent tensile fatigue damage. Bazant and Hubler's work showed that while tensile fatigue damage of concrete can be characterized by propagation of Mode I cracks, compressive fatigue damage of concrete can be characterized by propagation of Mode I crushing bands [23]. Inspired by their work, by considering compressive fatigue damage as fictitious crushing bands, we could obtain a similar effective CO₂ diffusion coefficient for concrete with compressive fatigue damage as compared with that for concrete with tensile fatigue damage (see Eq. (2)). In other words, effective CO₂ diffusion coefficient for concrete with tensile or compressive fatigue damage can be rewritten into a generalized form, as shown by Eq. (2), where the total effective CO₂ diffusion coefficient is composed of a non-damaged concrete contribution term and a fatigue damage contribution term:

$$D_d = \underbrace{k_0 \cdot \varphi_p^{k_1} \cdot (1 - RH)^{k_2} \cdot (k_3 \cdot T + k_4)}_{\text{Contribution by Non-damaged Concrete}} + \underbrace{|\varepsilon_r(x)| \cdot (1 - \alpha \cdot \lambda_1(x)) \cdot D_{\text{air}}}_{\text{Contribution by Fatigue Damage}} \quad (2)$$

where, k_0 , k_1 , k_2 , k_3 , and k_4 are fitting coefficients; $\varepsilon_r(x)$ is the space-dependent residual strain; $\lambda_1(x)$ denotes the aggregate fraction at each concrete fiber; φ_p is the porosity of hardened cement paste; α is an empirical coefficient, which is taken as 0 and 0.5 for tensile and compressive damage, respectively; D_{air} is the CO₂ diffusion coefficient in air, which can be calculated by the Wille-Lee or Fuller et al.'s method (see Ref. [15] for details). The fitting coefficients k_0 , k_1 and k_2 are equal to 1.64×10^{-6} , 1.8 and 2.2, respectively. These fitting coefficients apply to ordinary Portland cement-based concrete with water-cement ratios ranging from 0.5 to 0.8 [22]. For concrete mixed with ordinary Portland cement and supplementary cementing materials, in which water-binder ratios vary from 0.38 to 0.58, k_0 , k_1 and k_2 are equal to 6.10×10^{-6} , 3 and 2.2, respectively [24]. The fitting coefficients k_3 and k_4 should be equal to 0.02 and 0.6, respectively [25]. The porosity of hardened cement paste, φ_p , varies with the degrees of both hydration and carbonation reactions, which can be calculated by the formulas proposed by Papadakis et al. [22]. Residual strain $\varepsilon_r(x)$ can be calculated as (see Fig. 1c):

$$\varepsilon_r(x) = \begin{cases} \varepsilon_{\text{rm}} & (\text{UCD}) \\ \varepsilon_{\text{rm}}^t - \kappa_r \cdot x & (\text{GTD}) \\ \varepsilon_{\text{rm}}^c + \kappa_r \cdot x & (\text{GCD}) \end{cases} \quad (3)$$

in which, ε_{rm} , $\varepsilon_{\text{rm}}^t$, and $\varepsilon_{\text{rm}}^c$ are residual strains at the edges of UCD, GTD and GCD concrete zones, respectively. These residual strains can be calculated by the fatigue damage prediction model proposed by the authors [26]; κ_r represents the residual curvature which can also be calculated by the fatigue damage prediction model [26]. Different from our previous work [15] in which λ_1 was considered as a constant, the NCM in this paper considered λ_1 as a space-dependent parameter in order to take into account the wall effect, as shown by Eq. (4):

$$\lambda_1(x) = \begin{cases} \lambda_{\text{co}} \cdot x / (a_{\text{max}}/2) & 0 \leq x \leq a_{\text{max}}/2 \\ \lambda_{\text{co}} & a_{\text{max}}/2 \leq x \leq L_x - a_{\text{max}}/2 \\ \lambda_{\text{co}} \cdot (L_x - x) / (a_{\text{max}}/2) & L_x - a_{\text{max}}/2 \leq x \leq L_x \end{cases} \quad (4)$$

where, a_{max} is the maximum size of aggregates, L_x denotes the dimension of the specimen in x direction, and λ_{co} is the aggregate fraction in the core area.

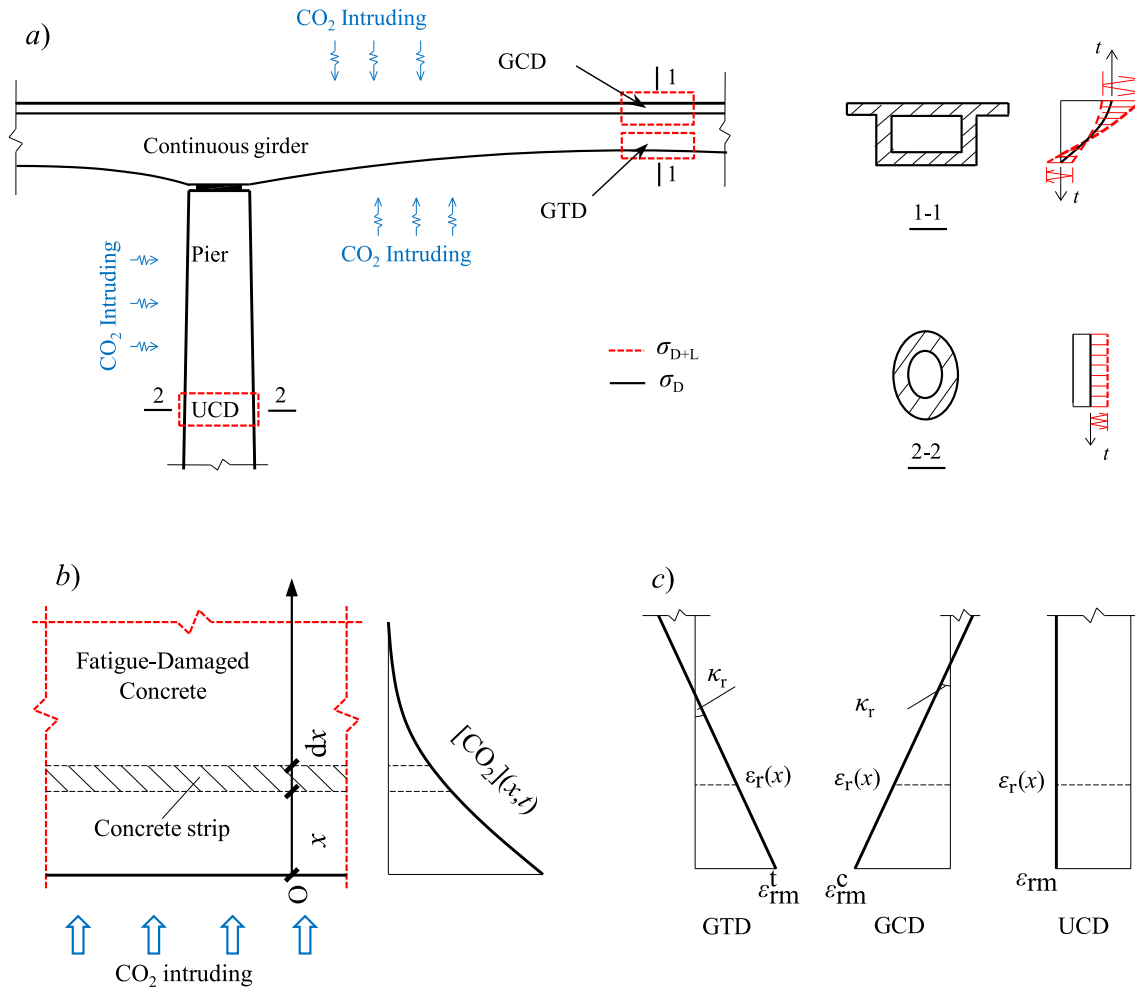


Fig. 1. Modeling carbonation process in fatigue-damaged concrete: a) concrete in a bridge subjected to fatigue loading as well as CO₂ erosion; b) carbon dioxide concentration profile in UCD, GCD, or GTD concrete at a certain time t ; c) fatigue damage states of concrete with UCD, GCD and GTD patterns. For simplicity, the zero point of the x coordinate is always set at the edge of the UCD, GTD or GTD concrete zone, where fatigue damage is at its highest degree.

In summary, D_d is determined as a function of exposure conditions (RH and T), the material composition of concrete (embedded in φ_p), and the space-dependent fatigue damage state.

2.3. Total carbonation reaction rate (Q)

Fatigue damage does not alter the material compositions of concrete, but it may change the volume of a concrete specimen and volumetric changes can vary in space. This may further yield space-dependent reaction rates because concentrations of carbonatable constituents theoretically should be based on the volume of a concrete specimen after fatigue damage. However, because volumetric strain is very small in general, it is reasonable to use original concrete volume to calculate concentrations of carbonatable constituents in fatigue-damaged concrete; hence, effects of fatigue damage on carbonation reaction rates are negligible. The total reaction rate of carbonation in fatigue-damaged concrete is thus regarded as the same as that in non-damaged concrete. According to the experimental results reported by Papadakis et al. [27], carbonatable constituents of concrete include calcium hydroxide ($\text{Ca}(\text{OH})_2$ denoted as CH), calcium silicate hydrate ($3\text{CaO} \cdot 2\text{SiO}_2 \cdot 3\text{H}_2\text{O}$ denoted as CSH), unhydrated tricalcium silicate ($3\text{CaO} \cdot \text{SiO}_2$ denoted as C_3S) and dicalcium silicate ($2\text{CaO} \cdot \text{SiO}_2$ denoted as C_2S). Therefore, the total carbonation rate is the sum of four parts [27]:

$$Q = R_{\text{CH}} + 3R_{\text{CSH}} + 3R_{\text{C}_3\text{S}} + 2R_{\text{C}_2\text{S}} \quad (5)$$

where R_i ($i = \text{CH}, \text{CSH}, \text{C}_3\text{S}$ and C_2S) represents the reaction rate of CO₂ with CH, CSH, C_3S , or C_2S , respectively, and can be calculated by Eq. (6) [27].

$$R_i = K_i \cdot [i] \cdot [\text{CO}_2] \quad (6)$$

in which $[i]$ is the molar concentration of the carbonatable constituent i (mol/m^3), and K_i is the corresponding carbonation reaction rate constant ($\text{m}^3/\text{mol} \cdot \text{s}$) (see Ref. [27] for details). Neglecting the hydration reactions during the carbonation process, the following equations hold true for mass balances of CH, CSH, C_3S and C_2S :

$$\frac{\partial [i]}{\partial t} = -K_i \cdot [i] \cdot [\text{CO}_2] \quad (7)$$

where, $i = \text{CH}, \text{CSH}, \text{C}_3\text{S}$ and C_2S .

Combining with specific boundary and initial conditions, the carbonation equation (Eq. (1)) is uniquely determined and can be numerically solved by a Keller Box Scheme [28]. The NCM inputs concrete material compositions, exposure conditions and fatigue damage states, and outputs time- and space-varying CH, CSH, C_3S , C_2S , pH as well as time-evolving carbonation depths where $\text{pH} = 9$. The numerical model was verified by our experimental results [15,18] and several other compatible experimental results in the literature [12,13,16,17], as presented in Section 3.

Table 1
Material compositions of collected concrete specimens.

Nos.	Mix design (kg/m ³)						Aggregates		[CH] ⁰	[CSH] ⁰	ϕ_{pc}
	m_w	m_c	m_{fa}	m_{ca}	m_{sa}	m_{sf}	a_{max} (mm)	ρ_{ca} (kg/m ³)	(mol/m ³)	(mol/m ³)	
ND1-4	191.71	361.72	0	1085.16	723.44	0	16	2628	1003.00	587.11	0.2869
AL5-14	191.71	361.72	0	1085.16	723.44	0	16	2628	1003.00	587.11	0.2869
EL5-14	191.71	361.72	0	1085.16	723.44	0	16	2628	1003.00	587.11	0.2869
B1-3	194.8	367.5	0	1102.6	735.1	0	20	2716	1048.50	636.57	0.2648
TC	180	300	0	777	1002	0	16	2636	1346.8	699.00	0.4250
TM	250	417	0	0	1554	0	/	/	968.90	502.88	0.5067
JJY0-2	164	388.8	97.2	956	806	62.4	16	2740	449.80	735.85	0.1320
HJD1-16	154	310	130	1120	685	0	20	2740	82.02	605.37	0.1161
ZYX0-4	170	350	90	1103	675	0	31.5	2740	190.93	677.85	0.2135

Notes: ND, AL and EL specimens are taken from Ref. [18]; B specimens are taken from Ref. [15]; TC and TM specimens are taken from Ref. [12]; JJY0-2 specimens are taken from Ref. [13]; HJD specimens are taken from Ref. [16]; ZYX specimens are taken from Ref. [17]. m_w , m_c , m_{fa} , m_{ca} , m_{sa} and m_{sf} are the mass contents of water, cement, fly ash, coarse aggregate, sand and steel fiber, respectively, in a unit volume of concrete. The italics in the table are those values not offered in the original sources but used in the model calculations in this paper.

3. Model validation

3.1. Summary of experiments

Although Wang [14] reported a group of carbonation depths in fatigue-damaged concretes, Wang's experiment was not chosen to validate our models because the original author did not specify whether each reported carbonation depth was the average one or maximum one on each tested cut or split surface sprayed with a phenolphthalein solution. The material compositions of concrete specimens collected from the literature [12,13,15–18] are summarized in Table 1. All the cements in these experiments are ordinary Portland cements without supplementary cementitious materials. Moreover, the initial concentrations of carbonatable substances and porosities of fully carbonated cement pastes of these concrete specimens are also shown in Table 1, which were calculated by the formulas proposed by Papadakis et al. [22] and Papadakis [24] based on the mix design of concrete, curing conditions and durations. Obviously, these concrete specimens exhibit noticeable variations of material compositions.

The fatigue load methods, exposure conditions and exposure durations of these collected concrete specimens are shown in Table 2. Among these experiments, the third-point bending fatigue test was the most widely used method to obtain fatigue-damaged specimens, which particularly yielded concrete specimens with representative GTD. Moreover, the uniaxial compression and eccentric compression fatigue tests were also utilized to achieve concrete specimens with representative UCD and GCD, respectively. In summary, these experiments involved three representative fatigue damage patterns commonly existing in real-world RC bridges. The exposure conditions and exposure durations of these collected specimens also displayed obvious

variations, as shown in Table 2.

3.2. Model predictions vs. experimental results

With basic information provided by several collected experiments from the literature, the carbonation depths under the experimental exposure conditions at the experimental exposure durations were calculated by the NCM. Fig. 2 compares the 344 carbonation depths predicted by NCM with those reported by these collected experiments. The mean absolute errors (MAEs) and correlation coefficients (R^2) between NCM-predicted and experimental carbonation depths are also shown in Fig. 2, which are 0.79 mm and 0.98, respectively. Hence, it is safe to conclude that the NCM-predicted carbonation depths were in good agreement with those test results from the literature-based experiments, which demonstrated the validity of the proposed NCM.

4. Simplified carbonation model (SCM)

4.1. Simplification procedure

The carbonation depth in fatigue-damaged concrete is decomposed into a non-damaged concrete contribution term and a fatigue damage contribution term, as shown by Eq. (8):

$$x_c = x_0 + \theta \cdot x_1 \quad (8)$$

where, x_0 is the carbonation depth contributed by non-damaged concrete, and θx_1 is the carbonation depth contributed by fatigue damage. To get an explicit carbonation model (t vs. x_c), the effects of the fatigue damage gradient were not explicitly involved in the simplified carbonation model Eq. (8). That is, x_1 denotes the carbonation depth

Table 2
Fatigue load methods, exposure conditions and exposure durations of collected concrete specimens.

Nos.	Size (mm)	Loading method	Exposure conditions	Exposure durations (days)
ND1-4	100 × 100 × 300	N/A	Standard:	3, 7, 14, 28, 49
AL5-14	100 × 100 × 300	Uniaxial compression	20 °C, 70% RH, 20% CO ₂	3, 7, 14, 28, 49
EL5-14	100 × 100 × 300	Eccentric compression	Variations:	7, 14, 28
			T: 10 °C, 20 °C, 35 °C	
			RH: 60%, 70%, 80%	
			CO ₂ : 1.5%, 10%, 20%	
B1-3	150 × 150 × 600	Third-point bending	20 °C, 82% RH, 20% CO ₂	14, 28, 49
TC	100 × 100 × 400	Third-point bending	20 °C, 60% RH, 20% CO ₂	60
TM	100 × 100 × 400	Third-point bending	20 °C, 60% RH, 20% CO ₂	60
JJY0	100 × 100 × 300	N/A	20 °C, 70% RH, 20% CO ₂	7, 14, 28
JJY1	100 × 100 × 300	Uniaxial compression	20 °C, 70% RH, 20% CO ₂	7, 14, 28
JJY2	100 × 100 × 400	Third-point bending	20 °C, 70% RH, 20% CO ₂	7, 14, 28
HJD1-16	100 × 100 × 400	Third-point bending	20 °C, 70% RH, 20% CO ₂	3, 7, 14, 28
ZYX0-4	150 × 150 × 550	Third-point bending	20 °C, 70% RH, 20% CO ₂	7, 14, 28, 56

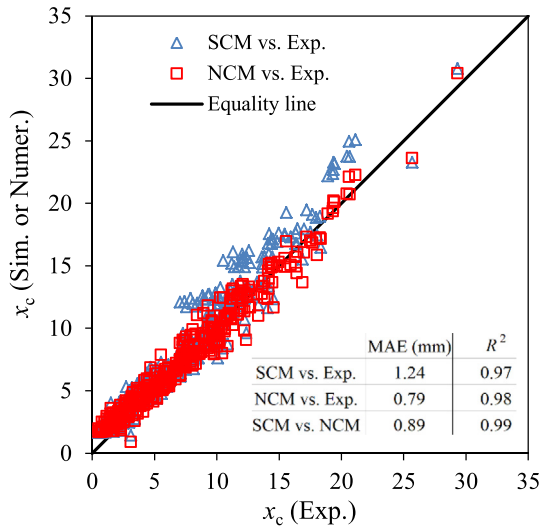


Fig. 2. Comparison of carbonation depths predicted by the SCM and NCM with experimental ones (unit: mm).

contributed by the concrete with effective CO_2 diffusivity equal to the second term on the right hand side of Eq. (2) while $x = 0$, i.e., with effective CO_2 diffusivity equal to $\varepsilon_r(0) \cdot D_{\text{air}}$. The influences of fatigue damage gradient are then embedded into the coefficient θ . x_0 and x_1 are carbonation depths in uniform concrete (fatigue-damaged and non-damaged); thus, they can be calculated by the Papadakis model [21] as:

$$x_0 = \sqrt{2 \left[k_0 \cdot \varphi_{\text{pc}}^{k_1} \cdot (1 - RH)^{k_2} \cdot (k_3 \cdot T + k_4) \right] \cdot m_0 \cdot t} \quad (9)$$

$$x_1 = \sqrt{2 [\varepsilon_r(0) \cdot D_{\text{air}}] \cdot m_0 \cdot t} \quad (10)$$

where, φ_{pc} is the porosity of fully carbonated cement paste, and m_0 is a parameter related to initial concentrations of carbonatable substances. Meanings of the remaining parameters are the same as those in Eq. (2). Thus, m_0 can be calculated as [21]:

$$m_0 = \frac{[\text{CO}_2]}{[\text{CH}]^0 + 3[\text{CSH}]^0} \quad (11)$$

in which, $[\text{CO}_2]$ is the molar concentration of CO_2 in the exposed environment. Therefore, $[\text{CH}]^0$ and $[\text{CSH}]^0$ are molar concentrations of calcium hydroxide ($\text{Ca}(\text{OH})_2$ denoted by CH) and calcium silicate hydrate ($3\text{CaO} \cdot 2\text{SiO}_2 \cdot 3\text{H}_2\text{O}$ denoted by CSH), respectively, in concrete at the beginning of carbonation.

For each real carbonation depth x_c in fatigue-damaged concrete, the corresponding θ value can be determined through substituting Eqs.(9) and (10) into Eq. (8). In such a way, if adequate real carbonation depths are collected, the distribution of θ values can be determined. However, the existing experimental carbonation depths in fatigue-damaged concrete are not comprehensive and systematic. Accordingly, if θ is calibrated by existing experimental results, its value may not be as representative as it should be in real cases. Considering the numerical carbonation model has been verified by existing experimental results, it is reasonable to assume the carbonation depth predicted by the numerical carbonation model as the “real” carbonation depth in the same concrete under the same exposure conditions at the same exposure duration. Consequently, given a certain concrete, through Monte Carlo simulations based on the numerical carbonation model, comprehensive and systematic “real” carbonation depths in fatigue-damaged concrete can be obtained. Correspondingly, representative θ values can be determined by the method mentioned at the beginning of this paragraph.

4.2. Monte Carlo simulations

Considering that material compositions of concrete can vary

Table 3

Mix designs of three different concrete grades.

Grades	Cement type	Mix proportion (kg/m ³)				Coarse aggregates		
		m_w	m_c	m_{ca}	m_{sa}	type	a_{max} (mm)	ρ_{ca} (kg/m ³)
C30	# 425 OPC	185.0	308.3	1105.2	736.8	Granite	16	2628
C40	# 425 OPC	185.0	370.0	1073.8	715.9	Granite	16	2628
C50	# 425 OPC	185.0	462.5	1026.6	684.4	Granite	16	2628

Note: m_w , m_c , m_{ca} and m_{sa} are the contents of water, cement, coarse aggregate and fine aggregates, respectively, in a unit volume of concrete; a_{max} and ρ_{ca} are the maximum particle size and density of coarse aggregates, respectively.

significantly with the designers' preferences for different needs or requirements, the material compositions of concrete were not regarded as explicit influencing factors in this paper. Instead, three commonly-used concretes, i.e., C30, C40 and C50, designed according to a Chinese standard [29] were reputed as prototype concretes, as shown in Table 3. C30, C40 or C50 means that the cubic strength of this concrete will not be lower than 30, 40 or 50 MPa at the curing age of 28 days with a guarantee rate of 95%. The initial concentrations of carbonatable substances in these concretes and the porosities of fully carbonated cement pastes in these concretes are shown in Table 4, which were calculated by the formulas proposed by Papadakis et al. [22] and Papadakis [24]. Given a certain concrete (C30, C40 or C50), each array, which is composed of six influencing factors: 1) residual strain at the concrete surface ($\varepsilon_r(0)$), 2) residual curvature (κ_r), 3) temperature (T), 4) relative humidity (RH), 5) CO_2 concentration and 6) exposure duration (t_{ex}), will yield a θ value. Therefore, through Monte Carlo simulations which randomly select 100 arrays of all six influencing factors from their respective value ranges, as shown in Table 5, a group of 100 θ values were obtained for each concrete (C30, C40 or C50) with either tensile or compressive fatigue damage. Moreover, sensitivities of θ to the six influencing factors were also analyzed. While conducting sensitivity analysis, each one of the six influencing factors was subject to change, and in the meantime the remaining five influencing factors were set at their standard values as shown in Table 5.

4.3. Sensitivities of θ to influencing factors

Fig. 3 illustrates the sensitivities of θ to the six influencing factors, in which θ_{st} denotes the θ value under standard values of $\varepsilon_r(0)$, κ_r , T , RH , CO_2 and t_{ex} shown in Table 5. Obviously, θ generally decreases with the increase of any one of the following factors: temperature, residual curvature, exposure duration and CO_2 concentration. While residual strain at the concrete surface increases, θ increases monotonously. The influence of relative humidity on θ is more complicated; i.e., θ increases preliminarily but decreases afterwards with the increase of relative humidity. Moreover, it is conceivable that the more closely the points of $\theta/\theta_{\text{st}}$ under an influencing factor approach a horizontal line, i.e., the smaller the range of $\theta/\theta_{\text{st}}$, the less sensitive is the θ value to that influencing factor. Fig. 3 also shows the range of $\theta/\theta_{\text{st}}$ under each influencing factor. Obviously, under the influences of temperature, residual curvature, relative humidity, CO_2 concentration, exposure duration, and residual strain at the concrete surface, the ranges

Table 4

Initial concentrations of carbonatable substances in hardened concrete and porosities in carbonated cement paste.

Grades	$[\text{CH}]^0$	$[\text{CSH}]^0$	φ_{pc}
C30	872.553 mol/m ³	521.803 mol/m ³	0.333
C40	1047.075 mol/m ³	626.170 mol/m ³	0.252
C50	1308.844 mol/m ³	782.712 mol/m ³	0.149

Table 5
Standard values and value ranges of influencing factors.

Influencing factors	Standard value	Value ranges
$ \varepsilon_r(0) $	0.001	0–0.002
κ_r	0.01/m	0–0.02/m
T	30 °C	5–55 °C
RH	75%	55%–95%
$CO_2\%$	10.015%	0.03%–20%
t_{ex}	100 days	0–200 day(s)

of θ/θ_{st} increase gradually. Correspondingly, the sensitivities of θ increase gradually to temperature, residual curvature, relative humidity, CO_2 concentration, exposure duration, and residual strain at the concrete surface.

4.4. Statistical characteristics of θ

Mean (μ_θ), standard deviation (σ_θ) and coefficient of variation (CV) of each group of θ values are summarized in Table 6. For each concrete (C30, C40 or C50), the mean of θ in the tensile damage case is always larger than that in the compressive damage case, but the coefficient of variation in the tensile damage case is usually smaller than that in the compressive damage case. Nevertheless, the standard deviations show negligible differences between the tensile and compressive damage cases for each concrete. While the means in either the tensile or compressive damage case increase with the increases of concrete strength from C30 to C50, the standard deviations and coefficients of variation do not show obvious increasing or decreasing correlations with the concrete strength.

Fig. 4 illustrates the histograms of each group of θ values, in which the total group represents the collections of all θ values in C30, C40 and C50 concretes with either tensile or compressive damage. Moreover, “(c)” or “(t)” (behind C30, C40, C50 and Total) denotes the compressive or tensile damage case, respectively. Fig. 5 compares the observed quantiles with the expected normal distribution quantiles. The closer

Table 6
Statistical characteristics of each group of θ values.

Grades	Compressive damage			Tensile damage		
	μ_θ	σ_θ	CV	μ_θ	σ_θ	CV
C30	0.427	0.137	32.1%	0.469	0.146	31.1%
C40	0.448	0.162	36.1%	0.495	0.169	34.1%
C50	0.530	0.144	27.2%	0.581	0.137	23.6%
Total	0.468	0.154	33.2%	0.515	0.159	31.0%

the points (observed θ , expected θ) are to the equality line, the higher the probability is that the observed group of θ values will follow a normal distribution. Visually, all these groups of θ values follow normal distributions. However, it necessitates more rigorous normal tests to examine the normality of each group of θ values. In this paper, the joint skewness (b_s) and kurtosis (b_k) tests, recommended by Ref. [30], were utilized.

To examine the normality of each group of θ values, the b_s and b_k values of that group were calculated by the formulas specified by Ref. [30], as shown in Table 7. Then, the points ($|b_k|$, b_s) were plotted on a diagram with envelopes determined by the level of significance (α) and the number (N) of data points, as illustrated by Fig. 6. If the point ($|b_k|$, b_s) of a group of θ values locates within the corresponding envelope determined by N and α , it is accepted that the group of θ values follows a normal distribution under this level of significance. As shown by Fig. 6, under the level of significance 0.05, the groups C30(c), C30(t), C40(c) and Total(c) follow normal distributions. Under the level of significance 0.01, the groups C30(c), C30(t), C40(c), Total(c) and Total (t) follow normal distributions.

4.5. Recommendations for θ in SCM

If the material composition of a certain concrete is known as that of C30, C40 or C50 concrete shown in Table 3, the θ values in the SCM Eq. (8) can be taken as the mean values of the C30, C40 or C50 group

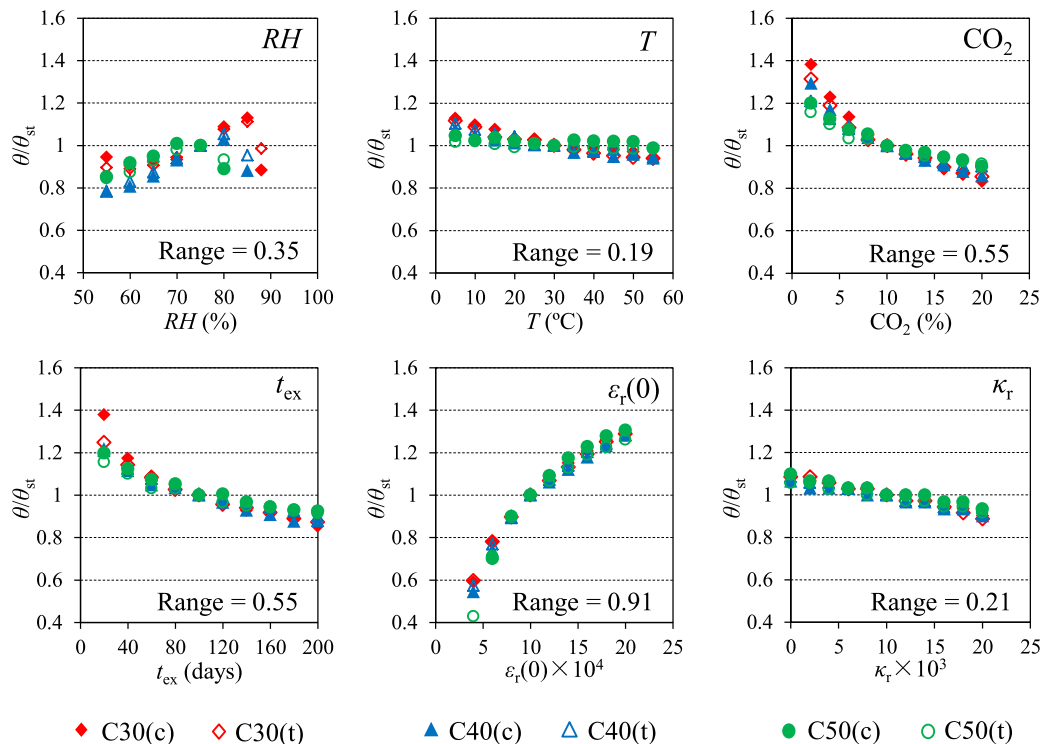


Fig. 3. Sensitivity of θ to each influencing factor.

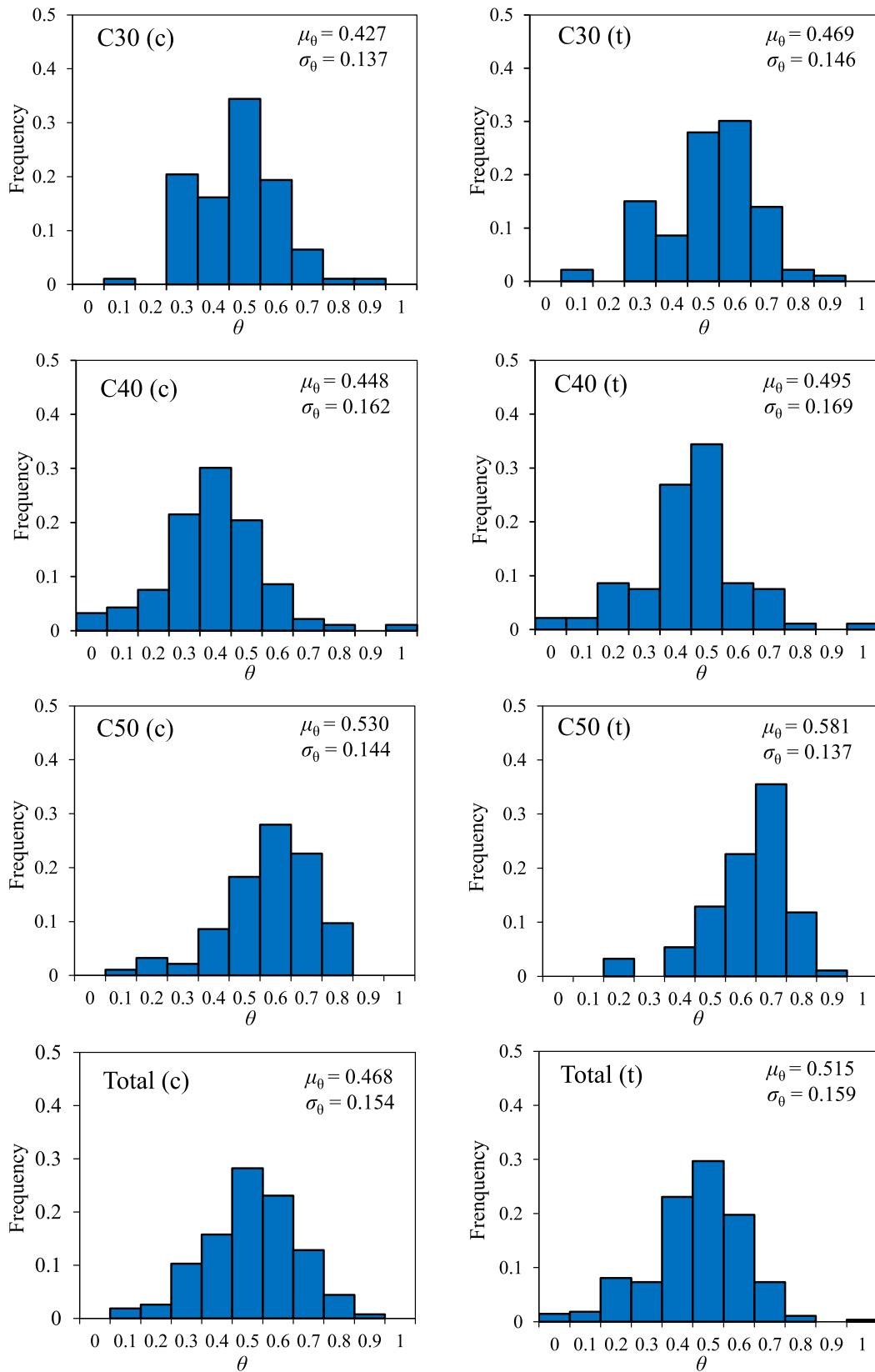


Fig. 4. Histograms of each group of θ values.

shown in Table 6, or can be taken as some quantile values to consider the reliability needs. In more common cases in which the material compositions of concretes are different from those shown in Table 6, it is recommended to take θ as the mean values of the Total group, or as

some quantile values of the Total group to meet the requirements for reliability. In this paper, the θ values in the SCM are taken as 0.468 and 0.515 for the compressive and tensile damage cases, respectively, which are the mean values of the Total group as shown in Table 6. The

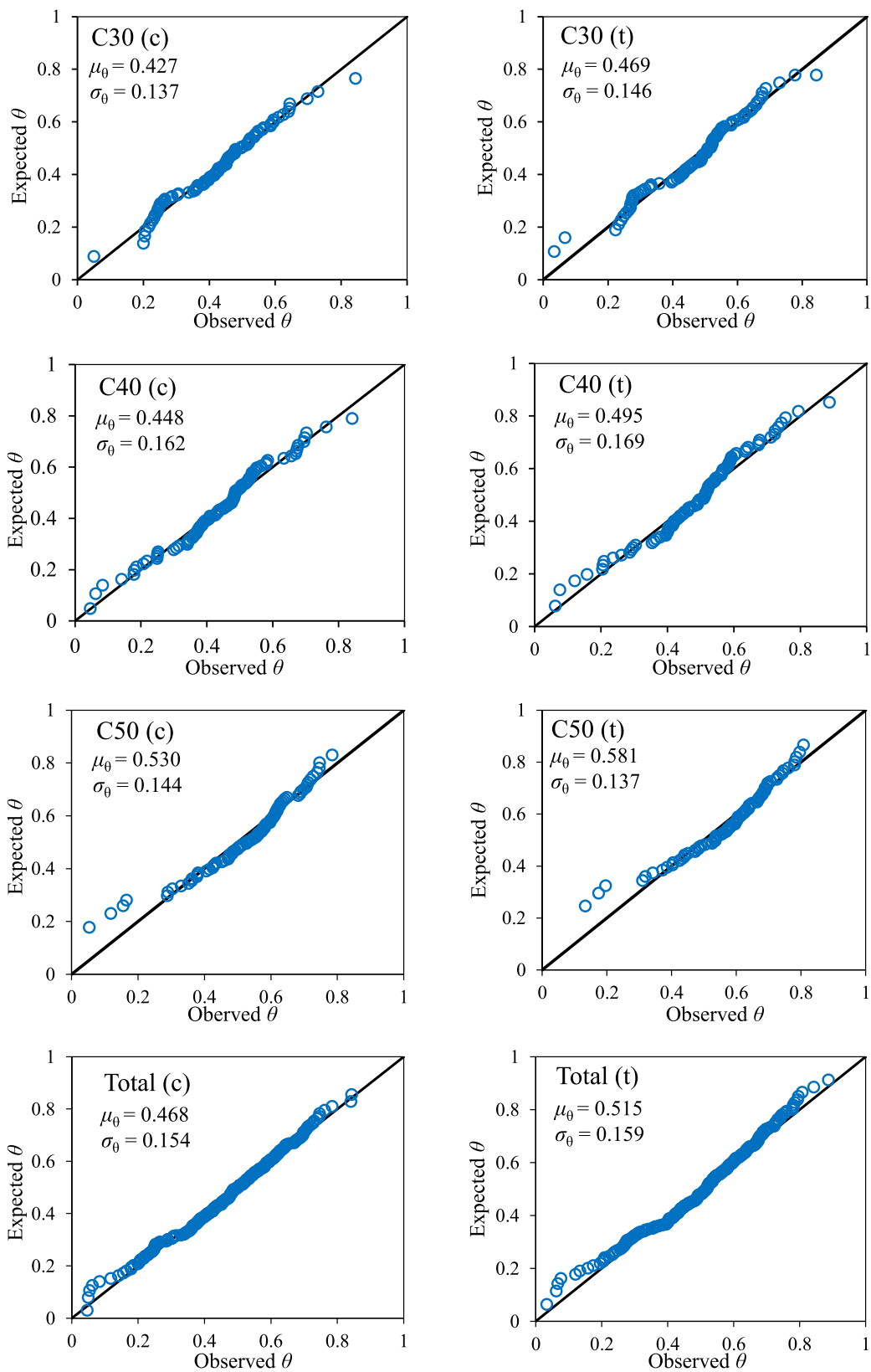


Fig. 5. Observed quantiles vs. expected normal distribution quantiles in each group of θ values.

Table 7
Skewness and kurtosis of each group of θ values.

Parameters	C30(t)	C30(c)	C40(t)	C40(c)	C50(t)	C50(c)	Total(t)	Total(c)
b_s	−0.3514	0.0674	0.2560	0.1878	−1.0001	−0.9762	−0.2496	−0.1534
b_k	3.3032	3.2052	5.2487	4.2324	4.1682	4.1604	3.9800	3.3771

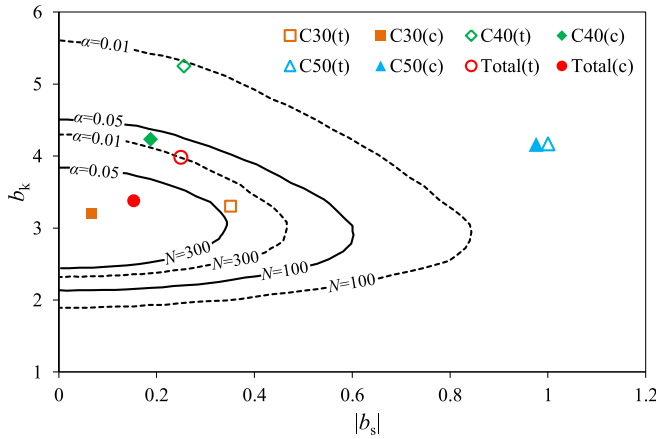


Fig. 6. Joint skewness and kurtosis tests for each group of θ values.

simplified carbonation model then can be rewritten into the following form as a function of exposure conditions RH , T , $[CO_2]$, and exposure duration t , as shown by Eq. (12).

$$x_c = f(RH, T, [CO_2], \varepsilon_r(0); t) = \underbrace{\left(\sqrt{k_0 \varphi_{pc}^{k_1} (1-RH)^{k_2} (k_3 T + k_4)} + \theta \cdot \sqrt{|\varepsilon_r(0)| \cdot D_{air}} \right)}_{\text{carbonation rate}} \cdot \sqrt{\frac{2[CO_2]}{[CH]^0 + 3[CSH]^0}} \cdot \sqrt{t} \quad (12)$$

In Fig. 2, the carbonation depths predicted by the recommended SCM were compared with the experimental and NCM-predicted ones. The high correlation coefficients and small MAEs in Fig. 2 demonstrated the validity of the SCM.

5. Incremental method

The simplified carbonation model Eq. (12) can be used to accurately predict carbonation depths in a certain concrete with determined fatigue damage under constant exposure conditions. However, as reported by the IPCC (Intergovernmental Panel on Climate Change) [19], due to burning of fossil fuels and deforestation of lands, CO_2 in the atmosphere continues to increase, which in turn increases the atmospheric temperature known as global warming. Moreover, fatigue damage of concrete bridges is accumulating with increased traffic and aging. In other words, the existing and planned concrete bridges will undergo time-variant exposure conditions and time-variant fatigue damage during their service life, as shown in Fig. 7. Therefore, Eq. (12) alone is not applicable to carbonation depth predictions in concrete bridges under these time-variant changes in the future. To consider the time-variant exposure conditions and fatigue damage, an incremental method based on Eq. (12) was introduced and is illustrated in Fig. 8.

In short, the incremental method calculates the carbonation depth $[x_c]_{n+1}$ at t_{n+1} based on the carbonation depth $[x_c]_n$ at t_n , as shown in Fig. 8. It is assumed that during each time interval, t_n to t_{n+1} , the exposure conditions (RH , T , CO_2) and fatigue damage ($\varepsilon_r(0)$) have achieved their respective average values ($RH_{n+1/2}$, $T_{n+1/2}$, $[CO_2]_{n+1/2}$ and $[\varepsilon_r(0)]_{n+1/2}$) between those values at t_n and t_{n+1} , as illustrated by Fig. 7. Hence, the carbonation depth evolution between t_n and t_{n+1} is controlled by Eq. (12) with $RH_{n+1/2}$, $T_{n+1/2}$, $[CO_2]_{n+1/2}$ and $[\varepsilon_r(0)]_{n+1/2}$, as follows:

$$x_c = f([CO_2]_{n+1/2}, RH_{n+1/2}, T_{n+1/2}, [\varepsilon_r(0)]_{n+1/2}; t) \quad (13)$$

However, at t_n , carbonation depth has reached $[x_c]_n$. Consequently, before calculating $[x_c]_{n+1}$, the equivalent time t_n^* required to reach $[x_c]_n$ under the constant exposure conditions of $RH_{n+1/2}$, $T_{n+1/2}$ and $[CO_2]_{n+1/2}$; moreover, the fatigue damage $[\varepsilon_r(0)]_{n+1/2}$ is calculated inversely by Eq. (13) as follows:

$$t_n^* = f^{-1}([CO_2]_{n+1/2}, RH_{n+1/2}, T_{n+1/2}, [\varepsilon_r(0)]_{n+1/2}; [x_c]_n) \quad (14)$$

Correspondingly, the carbonation depth $[x_c]_{n+1}$ at t_{n+1} can be calculated by the following equation as illustrated in Fig. 8:

$$[x_c]_{n+1} = f([CO_2]_{n+1/2}, RH_{n+1/2}, T_{n+1/2}, [\varepsilon_r(0)]_{n+1/2}; t_{n+1}^*) \quad (15)$$

in which, $t_{n+1}^* = t_n^* + \tau$ and $\tau = t_{n+1} - t_n$. In such a way, the carbonation depth predictions in concrete bridges under time-variant exposure conditions and cumulating fatigue damage can be addressed.

It is worth mentioning that the time interval τ should be long enough to assure that RH and T in the inner concrete during t_n and t_{n+1} have achieved the values $RH_{n+1/2}$ and $T_{n+1/2}$ in the ambient exposure environment. In this paper, τ is taken as one year. Correspondingly, annual mean values of RH and T in the exposure environment are used in calculations, neglecting seasonal variations of RH and T . This treatment has been widely adopted by researchers [31–33] who predicted carbonation depths factoring in global climate change. Although Talukdar et al. [34] calibrated the parameters on the effective diffusivity of CO_2 by carbonation tests with varying T , RH and CO_2 concentration conditions, these parameter values were not necessarily applicable to the carbonation process in a natural environment. This is because the variations of T , RH and CO_2 concentration in a natural environment are much more complex than those involved in the carbonation tests conducted by Talukdar et al. In fact, to identify the effects of variations of the natural exposure conditions on the carbonation process, it is required to solve a group of fully coupled partial differential equations governing heat, moisture and CO_2 transport in concrete with real-time monitored data of environmental actions as inputs, which is a project in progress in the authors' group. However, this is not presented here as it is beyond the scope of this paper.

Once the time-variant exposure conditions and cumulating fatigue damage are known, the carbonation depth at any point of an RC bridge during its service life can be predicted through implementing such an incremental method.

6. Case study

6.1. Project description

Fig. 9 illustrates the spans of a prestressed continuous RC girder and the dimensions of the mid-span (1-1) and pier (2-2) sections. Some geometry features are shown in Table 8. The material composition of concrete is: water: cement: coarse aggregate: fine aggregate = 0.5: 1: 2.9: 1.93. Meanwhile, $[CH]^0$, $[CSH]^0$ and φ_{pc} are 1047 mol/m^3 , 626 mol/m^3 and 0.252, respectively. The compressive strength of concrete is 38 N/mm^2 . The concrete tensile strength is taken as 10% of its compressive strength. The initial elastic modulus of concrete is $32,500 \text{ N/mm}^2$. The fatigue loads on the concrete part of the two cross-sections are shown in Table 8. In Table 8, P_D and M_D denote the axial force and the bending moment borne by the concrete part of each cross-section induced by dead loads (including the prestressing force); M_L is the bending moment borne by the concrete part of each cross-section

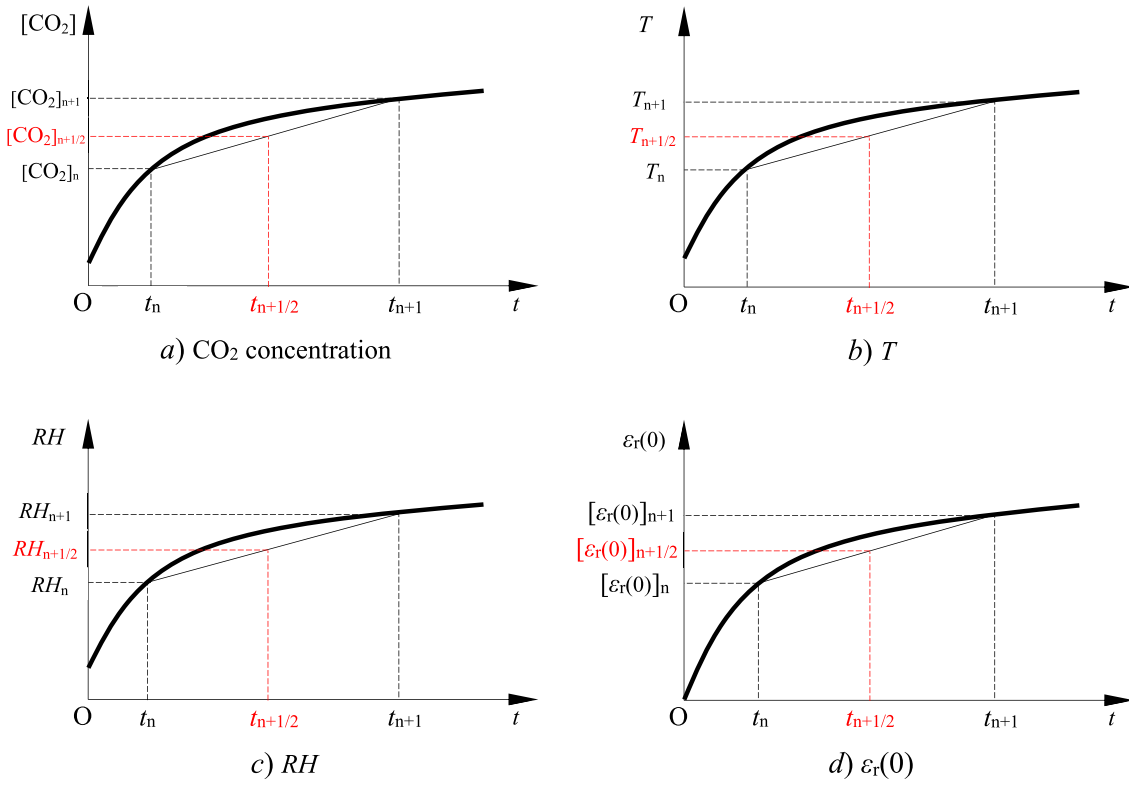


Fig. 7. Time-variant exposure conditions and cumulating fatigue damage.

induced by live loads. The initial stress distributions on the mid-span and pier cross-sections are also shown in Fig. 9.

Conceivably, even though traffic on roads is increasing, the maximum loads occurring during traffic jams generally do not increase very much because the weight of each vehicle and the number of vehicles occupying the road at one time are usually fairly constants. It was supposed that the fatigue loads on the concrete part of each cross section maintained their initial values as shown in Table 8 during its service life. Moreover, the effect of growing traffic volume usually has to do with an increase in the frequency of fatigue loads on concrete bridges. Hence, the loading frequency was set at 20,000 cycles per year so that the concrete bridge will have borne two-million cycles at the end of its anticipated service life.

The bridge was built in 2013 and designed for 100 years of service in Shanghai, China to support the high-speed train running there. The concrete cover thickness was designed as 35 mm according to Chinese standards [35–37]. This case study aims to predict the carbonation depth evolution at the O1, O2 and O3 locations of the concrete bridge (as shown in Fig. 9) under future changing climate conditions and

increasing traffic loads and simultaneously observe whether corrosion of steel reinforcements at these points will start by the year 2100.

6.2. Time-variant exposure conditions

IPCC has identified four representative concentration pathway (RCP) scenarios, i.e., RCP2.6, RCP4.5, RCP6 and RCP8.5, to describe future global climate changes [19]. RCP2.6 means that the radiative force, i.e., the difference of insolation (sunlight) absorbed by the Earth and energy radiated back to space, will reach 2.6 W/m² by the year 2100. This denotation rule also applies to the remaining three RCPs. Carbon dioxide concentrations from 2000 to 2100 under each RCP are shown in Fig. 10. Under RCP4.5, RCP6 and RCP8.5, carbon dioxide concentrations in the atmosphere will continue increasing. This excludes RCP2.6. Correspondingly, the temperature in the atmosphere will also continuously increase under RCP4.5, RCP6 and RCP8.5 except for RCP2.6. For example, Fig. 11 shows the temperature increases in Shanghai area by the year 2100 predicted by IPCC [19]. There is a dispute regarding the change of relative humidity in the atmosphere.

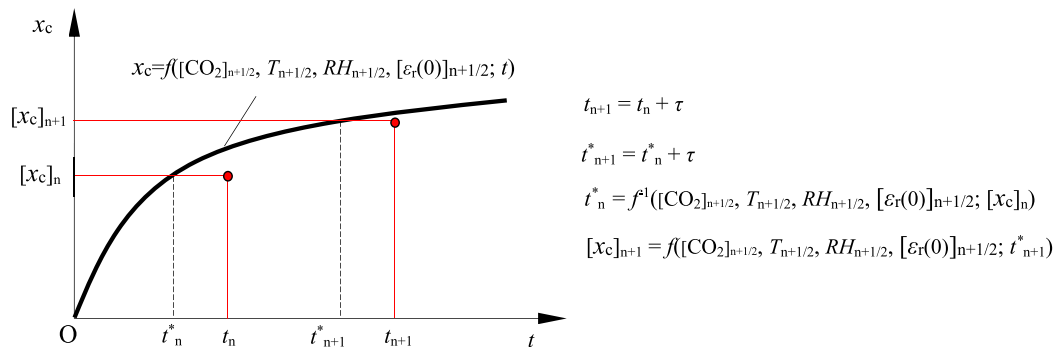


Fig. 8. Incremental method to calculate carbonation depths with time-variant $[CO_2]$, T , RH and $\varepsilon_r(0)$.

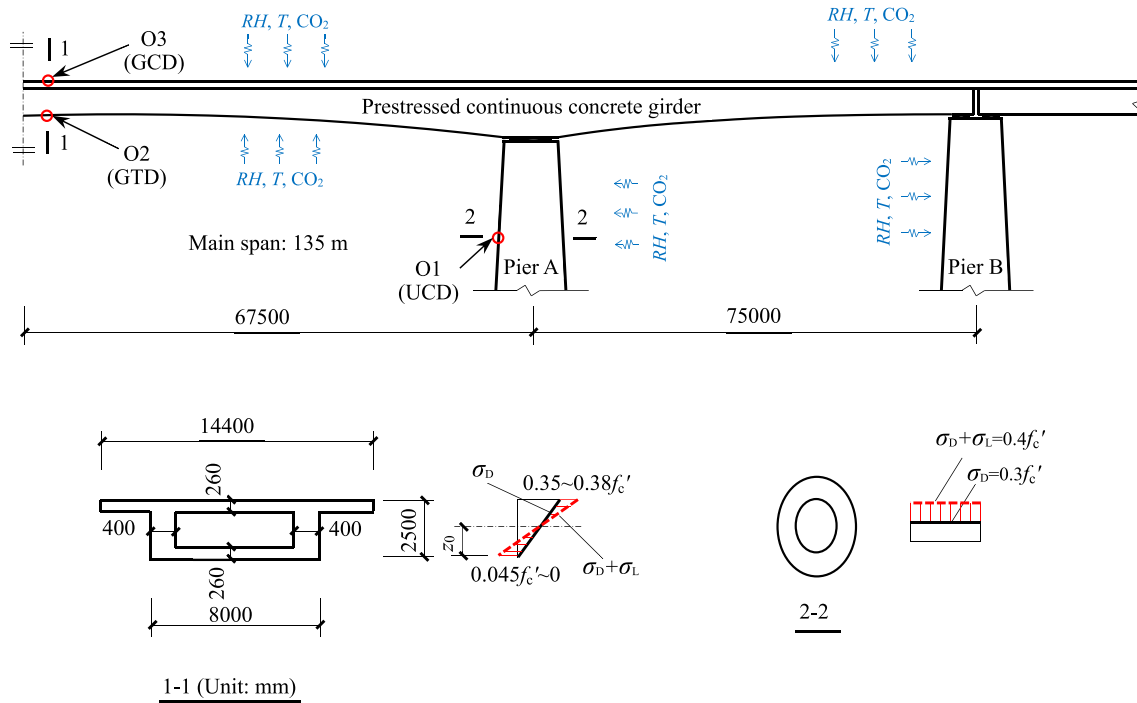


Fig. 9. Basic information about a representative concrete viaduct in high-speed railway network of China.

Table 8

Geometric features and loads borne by the concrete part of each section.

Section	A (mm ²)	I (mm ⁴)	c _b (mm)	P _D (kN)	P _L (kN)	M _D (kN m)	M _L (kN m)
1-1	7408000	7.3871 × 10 ¹²	1501.547	59115.840	0	39257.940	8412.416
2-2	659734	/	/	7520.972	2506.990	/	/

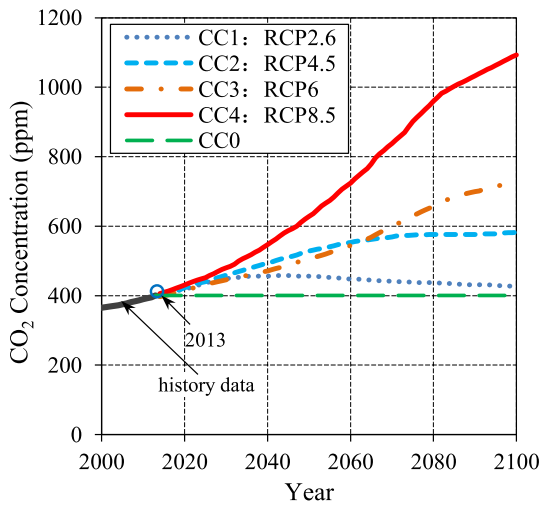


Fig. 10. CO₂ concentrations (ppm) under RCPs.

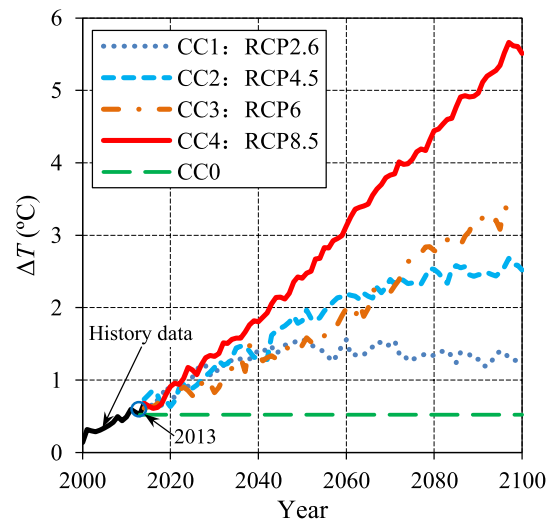


Fig. 11. Future temperature increases in Shanghai under RCPs.

Theoretically, global warming will increase the amount of moisture evaporating from the ocean to the atmosphere. However, according to field monitoring data, although global warming continues, no significant increase of annual relative humidity has been found [38]. Hence, IPCC did not report the changes of atmospheric relative humidity in its 2013 report [19]. On the other hand, not all researchers accepted the invariant relative humidity assumption. In fact, some researchers [39,40] thought relative humidity would increase 0–2.5% per decade. In this paper, it is accepted that global climate change does not change the annual relative humidity. Moreover, for convenience, five

climate conditions (CCs), i.e., CC0–CC5, are defined as illustrated by Fig. 10. CC0 means the exposure conditions will be the same as those in the year 2013, and CC1, CC2, CC3, and CC4 correspond to RCP2.6, RCP4.5, RCP6 and RCP8.5, respectively.

CO₂ concentrations around the concrete bridge are shown in Fig. 10. The temperature increases in Shanghai by the year 2100 are shown in Fig. 11. The annual mean temperature in Shanghai in the year 2013 is 15.32 °C. Combining the 2013 annual mean temperature and the temperature increases, the annual mean temperatures in Shanghai from

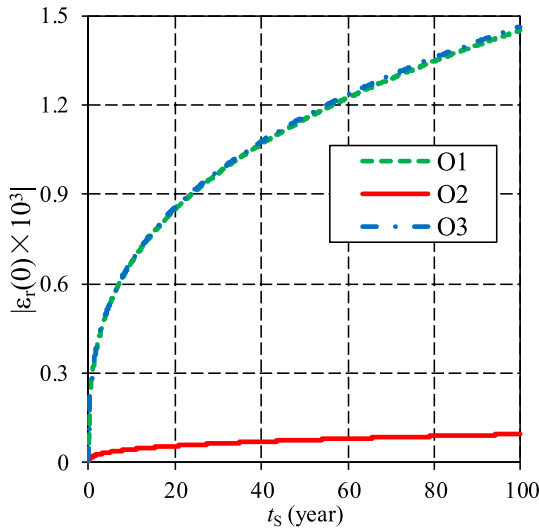


Fig. 12. Cumulating fatigue damage at O1, O2 and O3.

2013 to 2100 can be calculated. Moreover, the relative humidity around the concrete bridge is reputed as constant at 71.32%, which was the annual mean value in Shanghai in 2013.

6.3. Cumulating fatigue damage

The authors have proposed a fatigue deformation prediction model which can be used to predict strain and curvature evolution of a concrete structure during its service life [26]. Through inputting fatigue loads, cross-section parameters, concrete properties and expected cycles or times, the residual strains at O1, O2 and O3 were predicted by the fatigue deformation prediction model, as illustrated by Fig. 12. Obviously, during the service life, the residual strains at O1, O2 and O3 will increase monotonously. Since the magnitudes and frequencies of fatigue stresses at O1 and O3 are close to each other, the cumulating fatigue damage (the time-variant residual strain) at O1 and O3 are also close to each other. On the other hand, as the fatigue stresses at O2 are significantly different from those at O1 or O3, the cumulating fatigue damage (the time-variant residual strain) at O2 also noticeably deviates from those at O1 or O3. More details about the fatigue deformation prediction model can be found in Ref. [26].

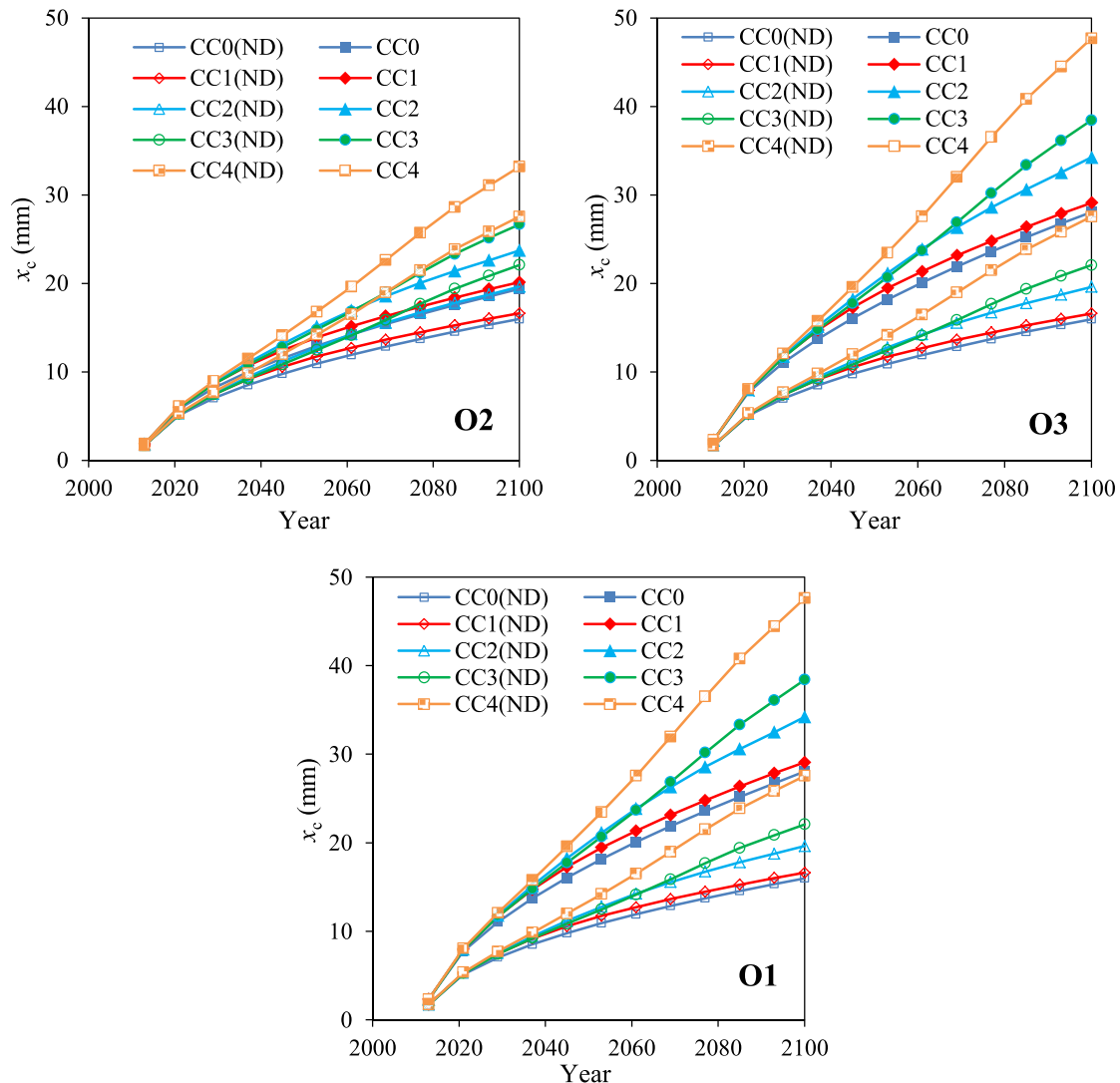


Fig. 13. Carbonation depth evolution at O1, O2 and O3.

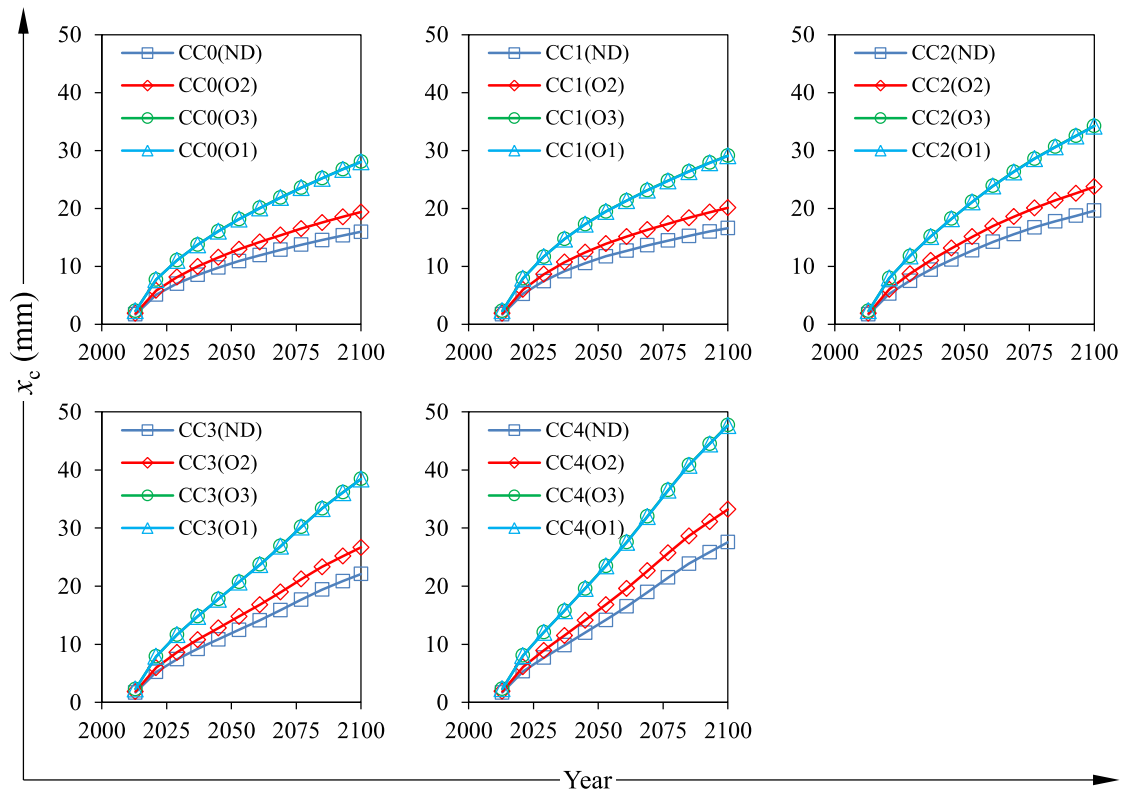


Fig. 14. Carbonation depth evolution under each climate change scenario.

Table 9

Times predicted for corrosion initiation (years).

Scenario	CC0	CC1	CC2	CC3	CC4
ND	> 88	> 88	> 88	> 88	> 88
O1	> 88	> 88	> 88	78	63
O2	> 88	> 88	> 88	> 88	> 88
O3	> 88	> 88	> 88	77	62

6.4. Carbonation depth predictions

Once the time-variant fatigue damage and exposure conditions are known, the carbonation depths in the RC bridge can be calculated by the proposed incremental method. Fig. 13 shows the carbonation depth evolution at O1, O2 and O3, in which “ND” denotes concrete without fatigue damage. Obviously, fatigue damage can make a big difference in the carbonation depth evolution of any one of O1, O2 and O3 location points under each climate condition (CC0, CC1, CC2, CC3 or CC4). Moreover, the carbonation depth evolution with service time at each point (O1, O2 or O3) under CC0 is always slower than it would be under CC1, CC2, CC3 or CC4; i.e., climate changes will always hasten carbonation depth development for all RC bridges in the future. In addition, due to drastic increases of CO₂ concentration and temperature under CC2, CC3 or CC4, the proportional relationship between the carbonation depth and the corresponding square root of service time will no longer be valid. Because the carbonation depth profile with respect to the service time under CC2, CC3 or CC4 is remarkably different from that under CC0 which is deemed to follow the proportional square root law. Hence, caution should be taken while using the coefficient of the proportional law, i.e., the carbonation rate shown in Eq. (12), as the carbonation resistance of a certain concrete can change under the context of global climate change in the future.

Fig. 14 also shows the carbonation depth evolution with service times at different points of the RC bridge under climate conditions, in

which “ND” denotes concrete without fatigue damage. Obviously, even though concrete material composition is the same, the carbonation depth evolution at O1, O2 or O3 differs from one another under each climate condition (CC0, CC1, CC2, CC3 or CC4). These differences can be attributed to the differences of the corresponding cumulating fatigue damage. As the cumulating fatigue damage history is very close between points O1 and O3 (as shown in Fig. 12), the carbonation depth evolution at point O1 is very close to that at O3 under each climate condition, as illustrated by Fig. 11. Otherwise, due to the noticeable differences of the cumulating fatigue damage between O1 and O2 (or O3), the carbonation depth evolution at O1 significantly deviates from that at O2 (or O3). This finding is consistent with the field test results which showed that carbonation depths differed remarkably from one another at different points in a 57-year-old highway bridge in Brazil [41].

Table 9 summarizes the different times at which the carbonation depth reaches steel reinforcements at each point of the RC bridge under different climate conditions. Obviously, if the cumulating fatigue damage is not considered, steel reinforcements at all points of the RC bridge will not start corrosion by the year 2100. If the cumulating fatigue damage is considered, steel reinforcement corrosion at points O1 and O3 under either CC3 or CC4 will be initiated before the year 2100. In other words, the cumulating fatigue damage can have a big influence on the prediction accuracy of the different amounts of time it will take to initiate carbonation-induced corrosion.

7. Conclusions

In this paper, a numerical carbonation model which deals with carbonation in fatigue-damaged concrete was introduced. A simplified carbonation model in fatigue-damaged concrete was then established, which was composed of a non-damaged concrete contribution term and a fatigue damage contribution term. Further, through comprehensive Monte Carlo simulations, the statistical characteristics of the values of

the coefficient θ of the fatigue damage contribution term were identified for commonly used C30, C40 and C50 concretes. It was found that the sensitivities of the coefficient θ increases gradually to temperature, residual curvature, relative humidity, CO₂ concentration, exposure duration, and residual strain at the concrete surface. Moreover, under the level of significance 0.01, the total group of θ values in either the tensile or compressive damage case follows a normal distribution. The mean value of the total group of θ values, i.e., 0.468 or 0.515 in the compressive or tensile damage case, was recommended in the proposed simplified carbonation model. Both the proposed SCM and NCM were verified by comparing their predicted carbonation depths with experimental carbonation depths reported in the literature. But it is worth mentioning that because both SCM and NCM were verified by carbonation experiments on ordinary Portland cement-based concretes, caution should be taken while applying them to supplementary cementitious materials-based concretes. In addition, considering these collected carbonation experiments were conducted in accelerated environments, much more research is needed to reconcile accelerated carbonation with natural carbonation. This is because carbonation of concrete under a natural environment differs, both physically and chemically, from that under an accelerated environment (see more detailed discussion in the Conclusion section of our previous paper [18]).

Subsequently, an incremental method was proposed to consider the time-variant fatigue damage and exposure conditions. Finally, a case study was conducted to observe the carbonation depth evolution up to the year 2100 at certain points of a representative RC bridge built in Shanghai, China to support high-speed train running there. The case study showed that the cumulating fatigue damage could pose a big influence on the carbonation depth evolution; meanwhile, noticeable differences in cumulating fatigue damage could lead to significant differences in carbonation depth evolution among the different location points in the RC bridge. These findings are consistent with field test results of carbonation depths in a 57-year-old highway concrete bridge in Brazil [41]. Moreover, the drastic changes of CO₂ concentration and temperature due to climate change could induce the relationships between carbonation depths and square roots of service times to remarkably deviate from the proportional laws widely accepted in constant natural or accelerated environments.

The findings in this case study serve as a warning to civil engineers that carbonation depth evolution in an RC bridge could be point-dependent on the stress history there, which could further lead to significant variations of onset of carbonation-induced corrosion in an RC bridge. Moreover, drastic climate changes could invalidate the proportional law between the carbonation depth and the square root of service time, which further questions the reasonability of carbonation rates, i.e., the coefficient of the proportional law, as the carbonation resistance of concrete in the future. Thus, civil engineers or designers must consider fatigue damage and climate change impacts on service life predictions or assessments of RC bridges in the future.

Acknowledgements

This research work was financially supported by the National Natural Science Foundation of China (Grant No. 51320105013) and the National Basic Research Program of China (973 Program) (Grant No. 2015CB655103). The first author appreciates the financial support of the International Postdoctoral Exchange Fellowship Program 2017 issued by the Office of China Postdoctoral Council (Approval No.: 32nd Doc. Of OCPC, 2017) while conducting revisions on this paper.

References

- [1] A. Solgaard, A. Michel, M. Geiker, H. Stang, Concrete cover cracking due to uniform reinforcement corrosion, *Mater. Struct.* 46 (2013) 1–19, <https://doi.org/10.1617/s11527-013-0016-6>.
- [2] L. Dai, L. Wang, J. Zhang, X. Zhang, A global model for corrosion-induced cracking in prestressed concrete structures, *Eng. Fail. Anal.* 62 (2016) 263–275, <https://doi.org/10.1016/j.engfailanal.2016.01.013>.
- [3] L. Jin, R. Zhang, X. Du, Y. Li, Investigation on the cracking behavior of concrete cover induced by corner located rebar corrosion, *Eng. Fail. Anal.* 52 (2015) 129–143, <https://doi.org/10.1016/j.engfailanal.2015.03.019>.
- [4] C.-Q. Li, J.J. Zheng, W. Lawanwisut, R.E. Melchers, Concrete delamination caused by steel reinforcement corrosion, *J. Mater. Civ. Eng.* 19 (2007) 591–600, [https://doi.org/10.1061/\(ASCE\)0899-1561\(2007\)19:7\(591\)](https://doi.org/10.1061/(ASCE)0899-1561(2007)19:7(591)).
- [5] A. Xu, A. Shayan, Relationship between reinforcing bar corrosion and concrete cracking, *ACI Mater. J.* 113 (2016) 3–12, <https://doi.org/10.14359/51688460>.
- [6] C. Lu, W. Jin, R. Liu, Reinforcement corrosion-induced cover cracking and its time prediction for reinforced concrete structures, *Corrosion Sci.* 53 (2011) 1337–1347, <https://doi.org/10.1016/j.corsci.2010.12.026>.
- [7] Y. Zhao, J. Yu, W. Jin, Damage analysis and cracking model of reinforced concrete structures with rebar corrosion, *Corrosion Sci.* 53 (2011) 3388–3397, <https://doi.org/10.1016/j.corsci.2011.06.018>.
- [8] W. Zhang, B. Zhou, X. Gu, H. Dai, Probability distribution model for cross-sectional area of corroded reinforcing steel bars, *J. Mater. Civ. Eng.* 26 (2014) 822–832, [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0000888](https://doi.org/10.1061/(ASCE)MT.1943-5533.0000888).
- [9] C.A. Apostolopoulos, M.P. Papadopoulos, Tensile and low cycle fatigue behavior of corroded reinforcing steel bars S400, *Construct. Build. Mater.* 21 (2007) 855–864, <https://doi.org/10.1016/j.conbuildmat.2005.12.012>.
- [10] W. Zhang, X. Song, X. Gu, S. Li, Tensile and fatigue behavior of corroded rebars, *Construct. Build. Mater.* 34 (2012) 409–417, <https://doi.org/10.1016/j.conbuildmat.2012.02.071>.
- [11] C.A. Apostolopoulos, Mechanical behavior of corroded reinforcing steel bars S500s tempcore under low cycle fatigue, *Construct. Build. Mater.* 21 (2007) 1447–1456, <https://doi.org/10.1016/j.conbuildmat.2006.07.008>.
- [12] K. Tanaka, J.H. Jeon, T. Nawa, H. Hashida, Effect of repeated load on micro structure and carbonation of concrete and mortar, in: M.A. Lacasse, D.J. Vanier (Eds.), *Durab. Build. Mater. Components 8 Serv. Life Asset Manag.*, NRC Research Press, Ottawa, 1999, pp. 256–265.
- [13] J. Jiang, W. Sun, J. Liu, J. Wang, C. Chen, Fatigue durability of steel fiber reinforced concrete with super-high vertical pumping distance, *J. Southeast Univ.* 36 (2006) 259–262 in Chinese.
- [14] J. Wang, Effects of Fatigue Damage and Cracks on concrete Durability, *Southeast University*, 2008, <https://doi.org/10.7666/d.y1244357> in Chinese.
- [15] C. Jiang, X. Gu, W. Zhang, W. Zou, Modeling of carbonation in tensile zone of plain concrete beams damaged by cyclic loading, *Construct. Build. Mater.* 77 (2015) 479–488, <https://doi.org/10.1016/j.conbuildmat.2014.12.088>.
- [16] J. Han, W. Liu, S. Wang, D.S. Geert, W. Sun, Y. Liang, Carbonation reaction and microstructural changes of metro-tunnel segment concrete coupled with static and fatigue load, *J. Mater. Civ. Eng.* 4016216 (2016) 1–11, [https://doi.org/10.1061/\(ASCE\)MT.1943-5533](https://doi.org/10.1061/(ASCE)MT.1943-5533).
- [17] Y. Zhou, D. Niu, Y. Miao, X. Lu, An experimental study of carbonation of concrete under flexural fatigue, *Ind. Constr.* 46 (2016) 123–127 in Chinese.
- [18] C. Jiang, Q. Huang, X. Gu, W. Zhang, Experimental investigation on carbonation in fatigue-damaged concrete, *Cement Concr. Res.* 99 (2017) 38–52, <https://doi.org/10.1016/j.cemconres.2017.04.019>.
- [19] IPCC (Intergovernmental Panel on Climate Change), *Climate Change 2013: the Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change*, Cambridge University Press, Cambridge, U.K., 2013.
- [20] T. Guo, D.M. Frangopol, D. Han, Y. Chen, Probabilistic assessment of deteriorating prestressed concrete Box-Girder bridges under increased vehicle loads and aggressive environment, *J. Perform. Constr. Facil.* 25 (2011) 564–576, [https://doi.org/10.1061/\(ASCE\)CF.1943-5509.0000174](https://doi.org/10.1061/(ASCE)CF.1943-5509.0000174).
- [21] V.G. Papadakis, C.G. Vayenas, M.N. Fardis, Fundamental modeling and experimental investigation of concrete carbonation, *ACI Mater. J.* 88 (1991) 363–373, <https://doi.org/10.14359/1863>.
- [22] V. Papadakis, C. Vayenas, M. Fardis, Physical and chemical characteristics affecting the durability of concrete, *ACI Mater. J.* 8 (1991) 186–196.
- [23] Z.P. Bazant, M.H. Hubler, Theory of cyclic creep of concrete based on Paris law for fatigue growth of subcritical microcracks, *J. Mech. Phys. Solid.* 63 (2014) 187–200, <https://doi.org/10.1016/j.jmps.2013.09.010>.
- [24] V.G. Papadakis, Effect of supplementary cementing materials on concrete resistance against carbonation and chloride ingress, *Cement Concr. Res.* 30 (2000) 291–299, [https://doi.org/10.1016/S0008-8846\(99\)00249-5](https://doi.org/10.1016/S0008-8846(99)00249-5).
- [25] D. Niu, L. Chen, C. Zhang, Calculating Model for Gas Diffusivity in concrete, vol. 39, J. Xi'an Univ. Arch. Tech. (Natural Sci.), 2007, pp. 741–745, <https://doi.org/10.15986/j.1006-7930.2007.06.009>.
- [26] C. Jiang, X. Gu, Q. Huang, W. Zhang, Deformation of concrete under high-cycle fatigue loads in uniaxial and eccentric compression, *Construct. Build. Mater.* 141 (2017) 379–392, <https://doi.org/10.1016/j.conbuildmat.2017.03.023>.
- [27] V.G. Papadakis, C.G. Vayenas, M.N. Fardis, Experimental investigation and mathematical modeling of the concrete carbonation problem, *Chem. Eng. Sci.* 46 (1991) 1333–1338, [https://doi.org/10.1016/0009-2509\(91\)85060-B](https://doi.org/10.1016/0009-2509(91)85060-B).
- [28] H.B. Keller, A new difference scheme for parabolic problems, in: B. Hubbard (Ed.), *Numer. Solut. Partial Differ. Equations, II*, Academic Press, New York, 1971, pp. 327–350.
- [29] China Industry Standard, Specification for Mix Proportion Design of Ordinary concrete (JGJ55-2011), 2011th ed., China Architecture & Building Press, Beijing, 2011 in Chinese.
- [30] ISO (the International Organization for Standardization), Statistical Interpretation of Data-Tests for Departure from the normal Distribution, (1997) <http://www.iso>.

- org/iso/catalogue_detail.htm?csnumber=22506.
- [31] I.S. Yoon, O. Copuroğlu, K.B. Park, Effect of global climatic change on carbonation progress of concrete, *Atmos. Environ.* 41 (2007) 7274–7285, <https://doi.org/10.1016/j.atmosenv.2007.05.028>.
 - [32] L. Peng, M.G. Stewart, Climate change and corrosion damage risks for reinforced concrete infrastructure in China, *Struct. Infrastruct. Eng.* 12 (2016) 499–516, <https://doi.org/10.1080/15732479.2013.858270>.
 - [33] M.G. Stewart, X. Wang, M.N. Nguyen, Climate change impact and risks of concrete infrastructure deterioration, *Eng. Struct.* 33 (2011) 1326–1337, <https://doi.org/10.1016/j.engstruct.2011.01.010>.
 - [34] S. Talukdar, N. Banthia, J.R. Grace, Carbonation in concrete infrastructure in the context of global climate change - Part 1: experimental results and model development, *Cement Concr. Compos.* 34 (2012) 924–930, <https://doi.org/10.1016/j.cemconcomp.2012.04.011>.
 - [35] China Industry Standard, *General Specifications for Design of Highway Bridges and Culverts (JTG D60-2015)*, China Communications Press, Beijing, 2015 in Chinese.
 - [36] China Industry Standard, *Code for Design on Reinforced and Prestressed concrete Structure of Railway Bridge and Culvert (TB 10002.3-2005/J 462-2005)*, Ministry of Railway of China, Beijing, 2005 in Chinese.
 - [37] China National Standard, *Code for Design of concrete Structures (GB 50010-2010)*, China Architecture & Building Press, Beijing, 2010 in Chinese.
 - [38] S. Talukdar, N. Banthia, Carbonation in concrete infrastructure in the context of global climate change: model refinement and representative concentration pathway scenario evaluation, *J. Mater. Civ. Eng.* 4015178 (2015) 1–7, [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0001438](https://doi.org/10.1061/(ASCE)MT.1943-5533.0001438).
 - [39] F. Sperling, R. Washington, R. Whittaker, Future climate Change of the subtropical north atlantic: implications for the cloud forests of Tenerife, *Climatic Change* 65 (2004) 103–123, <https://doi.org/10.1023/B:CLJM.0000037488.33377.bf>.
 - [40] M. Vuille, R.S. Bradley, M. Werner, F. Keimig, 20th century climate change in the tropical andes: observations and model results, *Climatic Change* 59 (2003) 75–99, https://doi.org/10.1007/978-94-015-1252-7_5.
 - [41] R.A. Medeiros-Junior, M.G. Lima, R. Yazigi, M.H.F. Medeiros, Carbonation depth in 57 years old concrete structures, *Steel, Compos. Struct* 19 (2015) 953–966, <https://doi.org/10.12989/scs.2015.19.4.953>.